CHAPTER 6
STEEL DESIGN THEORY

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CHAPTER 6
STEEL DESIGN THEORY

6.1 INTRODUCTION

Steel has higher strength, ductility and toughness than many other structural materials such as concrete or wood, and thus makes a vital material for bridge structures. In this chapter, basic steel design concepts and requirements for I-sections specified in the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2012) and the *California Amendments* (Caltrans, 2014) for flexure, shear, compression, tension, fatigue, and serviceability and constructibility are discussed. Design considerations, procedure and example for steel plate girders will be presented in Chapter 9.

6.2 STRUCTURAL STEEL MATERIALS

AASHTO M 270 (Grade 36, 50, 50S, 50W, HPS 50W, HPS 70W and 100/100W) structural steels are commonly used for bridge structures. AASHTO material property standards differ from ASTM in notch toughness and weldability requirements. When these additional requirements are specified, ASTM A 709 steel is equivalent to AASHTO M 270 and is pre-qualified for use in welded steel bridges.

The use of ASTM A 709 Grade 50 for all structural steel, including flanges, webs, bearing stiffeners, intermediate stiffeners, cross frames, diaphragms and splice plates is preferred. The use of ASTM A 709 Grade 36 for secondary members will not reduce material unit costs. The use of ASTM A 709 Grade 100 or 100W steel is strongly discouraged. The hybrid section consisting of flanges with a higher yield strength than that of the web may be used to save materials and is becoming more promoted due to the new high performance steels. Using HPS 70W top and bottom flanges in negative moment regions and bottom flanges in positive moment regions and Grade 50 top flanges in positive moment regions, and Grade 50 for all webs may provide the most efficient hybrid girder.

The use of HPS (High Performance Steel) and weathering steel is encouraged if it is acceptable for the location. FHWA Technical Advisory T5140.22 (FHWA, 1989) provides guidelines on acceptable locations. In some situations, because of a particularly harsh environment, steel bridges must be painted. Although weathering steel will perform just as well as conventional steel in painted applications, it will not provide superior performance, and typically costs more than conventional steel. Therefore, specifying weathering steel in painted applications does not add value and should be avoided. HPS and weathering steel should not be used for the following conditions:

- The atmosphere contains concentrated corrosive industrial or chemical fumes.
- The steel is subject to heavy salt-water spray or salt-laden fog.

- The steel is in direct contact with timber decking, because timber retains moisture and may have been treated with corrosive preservatives.
- The steel is used for a low urban-area overcrossing that will create a tunnel-like configuration over a road on which deicing salt is used. In these situations, road spray from traffic under the bridge causes salt to accumulate on the steel.
- The location has inadequate air flow that does not allow adequate drying of the steel.
- The location has very high rainfall and humidity or there is constant wetness.
- There is low clearance (less than 8 to 10 ft) over stagnant or slow-moving waterways.

6.3 DESIGN LIMIT STATES

Steel girder bridges shall be designed to meet the requirements for all applicable limit states specified by AASHTO (2012) and *California Amendments* (Caltrans, 2014). For a typical steel girder bridges, Strength I and II, Service II, Fatigue and Constructibility are usually controlling limit states.

6.4 FLEXURE DESIGN

6.4.1 Design Requirements

The AASHTO 6.10 and its Appendices A6 and B6 provide a unified flexural design approach for steel I-girders. The provisions combine major-axis bending, minor-axis bending and torsion into an interaction design formula and are applicable to straight bridges, horizontally curved bridges, or bridges combining both straight and curved segments. The AASHTO flexural design interaction equations for the strength limit state are summarized in Table 6.4-1. Those equations provide an accurate linear approximation of the equivalent beam-column resistance with the flange lateral bending stress less than $0.6F_y$ as shown in Figure 6.4-1 (White and Grubb 2005).

Table 6.4-1 I-Section Flexural Design Equations (Strength Limit State)

Section Type		Design Equation
Composite Sections in Positive Flexure	Compact	$M_u + \frac{1}{3} f_l S_{xt} \leq \phi_f M_n$ (AASHTO 6.10.7.1.1-1)
	Noncompact	Compression flange $f_{bu} \leq \phi_f F_{nc}$ (AASHTO 6.10.7.2.1-1) Tension flange $f_{bu} + \frac{1}{3} f_l \leq \phi_f F_{nt}$ (AASHTO 6.10.7.2.1-2)
Composite Sections in Negative Flexure and Noncomposite Sections	Discretely braced	Compression flange $f_{bu} + \frac{1}{3} f_l \leq \phi_f F_{nc}$ (AASHTO 6.10.8.1.1-1) Tension flange $f_{bu} + \frac{1}{3} f_l \leq \phi_f F_{nt}$ (AASHTO 6.10.8.1.2-1)
	Continuously braced	$f_{bu} \leq \phi_f R_h F_{yf}$ (AASHTO 6.10.8.1.3-1)

f_{bu} = flange stress calculated without consideration of the flange lateral bending (ksi)
f_l = flange lateral bending stress (ksi)
F_{nc} = nominal flexural resistance of the compression flange (ksi)
F_{nt} = nominal flexural resistance of the tension flange (ksi)
F_{yf} = specified minimum yield strength of a flange (ksi)
M_u = bending moment about the major axis of the cross section (kip-in.)
M_n = nominal flexural resistance of the section (kip-in.)
ϕ_f = resistance factor for flexure = 1.0
R_h = hybrid factor
S_{xt} = elastic section modulus about the major axis of the section to the tension flange taken as M_{yt}/F_{yt} (in.^{3})

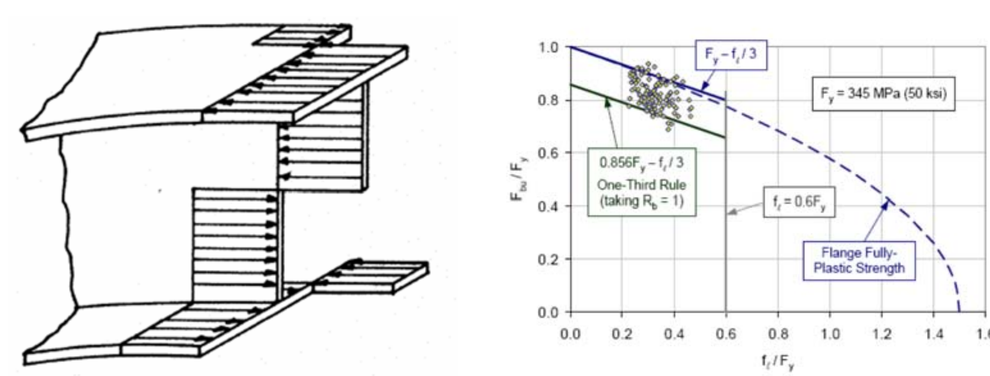


Figure 6.4 -1 AASHTO Unified Flexural Design Interaction Equations

For compact sections, since the nominal moment resistance is generally greater than the yield moment capacity, it is physically meaningful to design in terms of moment. For noncompact section, since the nominal resistance is limited to the yield strength, stress format is used. For composite I-sections in negative flexure and for noncomposite I-sections with compact or noncompact webs in straight bridges, when the web slenderness is well below the noncompact limit, the provisions specified in AASHTO Appendix 6A are encouraged to be used. However, when the web slenderness approaches the noncompact limit, Appendix 6A provides only minor increases in the nominal resistance.

6.4.2 Composite Sections in Positive Flexure

6.4.2.1 Nominal Flexural Resistance

At the strength limit state, the compression flange of composite sections in positive flexure is continuously supported by the concrete deck and lateral bending does not need to be considered. For compact sections, the flexural resistance is expressed in terms of moment, while for noncompact sections, the flexural resistance is expressed in terms of the elastically computed stress. The compact composite section shall meet the following requirements:

- Straight bridges
- $F_{yf} \leq 70\,\text{ksi}$
- $\dfrac{D}{t_w} \leq 150$ (AASHTO 6.10.2.1.1-1)
- $\dfrac{2D_{cp}}{t_w} \leq 3.76\sqrt{\dfrac{E}{F_{yc}}}$ (AASHTO 6.10.6.2.2-1)

where D_{cp} is the web depth in compression at the plastic moment (in.); E is modulus of elasticity of steel (ksi); F_{yc} is specified minimum yield strength of a compression flange (ksi). Composite sections in positive flexure not satisfying one or more of above four requirements are classified as noncompact sections. The nominal flexural resistances are listed in Table 6.4-2.

Table 6.4-2 Nominal Flexural Resistance for Composite Sections in Positive Flexure (Strength Limit State)

Section Type	Nominal Flexural Resistance
Compact	$$M_n = \min \begin{cases} M_p & \text{for } D_p \le 0.1D_t \\ M_p\left[1-\left(1-\frac{M_y}{M_p}\right)\left(\frac{D_p/D_t-0.1}{0.32}\right)\right] & \text{for } D_p > 0.1D_t \\ 1.3R_hM_y & \text{for a continous span} \end{cases}$$ (AASHTO 6.10.7.1.2-1, 3) and (CA 6.10.7.1.2-2)
Noncompact	Compression flange $F_{nc} = R_b R_h F_{yc}$ (AASHTO 6.10.7.2.2-1) Tension flange $F_{nt} = R_h F_{yt}$ (AASHTO 6.10.7.2.2-2)
Ductility Requirement	For both compact and noncompact sections $D_p \le 0.42D_t$ (AASHTO 6.10.7.3-1)

D_p = distance from the top of the concrete deck to the neutral axis of the composite section at the plastic moment (in.)
D_t = total depth of the composite section (in.)
F_{yt} = specified minimum yield strength of a tension flange (ksi)
M_p = plastic moment of the composite section (kip-in.)
M_y = yield moment of the composite section (kip-in.)
R_b = web load-shedding factor

6.4.2.2 Yield Moment

The yield moment M_y for a composite section in positive flexure is defined as the moment which causes the first yielding in one of the steel flanges. M_y is the sum of the moments applied separately to the appropriate sections, i.e., the steel section alone, the short-term composite section, and the long-term composite section. It is based on elastic section properties and can be expressed as:

$$M_y = M_{D1} + M_{D2} + M_{AD} \qquad \text{(AASHTO D6.2.2-2)}$$

where M_{D1} is moment due to factored permanent loads applied to the steel section alone (kip-in.); M_{D2} is moment due to factored permanent loads such as wearing surface and barriers applied to the long-term composite section (kip-in.); M_{AD} is additional live load moment to cause yielding in either steel flange applied to the short-term composite section and can be obtained from the following equation (kip-in.):

$$F_{yf} = \frac{M_{D1}}{S_{NC}} + \frac{M_{D2}}{S_{LT}} + \frac{M_{AD}}{S_{ST}} \qquad \text{(AASHTO D6.2.2-1)}$$

$$M_{AD} = S_{ST}\left[F_{yf} - \frac{M_{D1}}{S_{NC}} - \frac{M_{D2}}{S_{LT}}\right] \qquad (6.4\text{-}1)$$

where S_{NC}, S_{ST} and S_{LT} are elastic section modulus for steel section alone, short-term composite and long-term composite sections, respectively (in.^{3}).

6.4.2.3 Plastic Moment

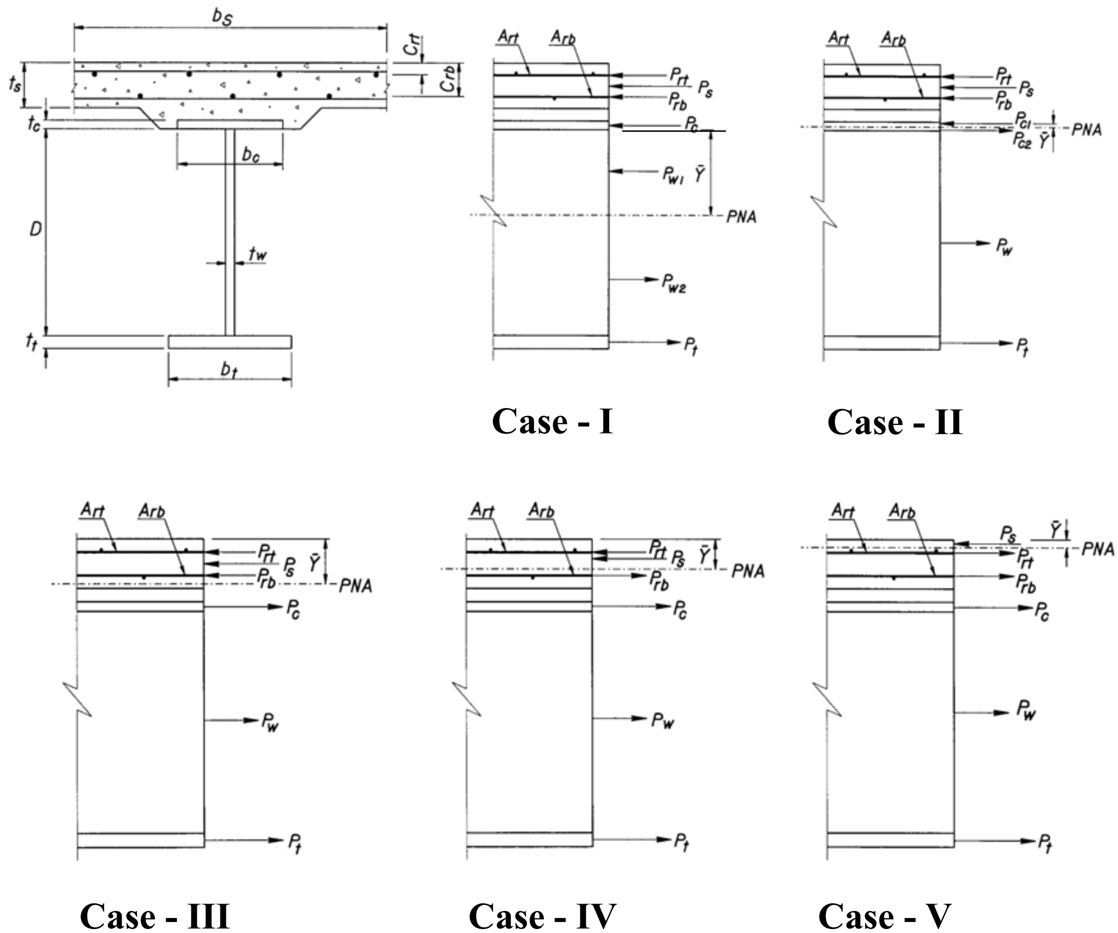
The plastic moment M_p for a composite section is defined as the moment which causes the yielding of the entire steel section and reinforcement and a uniform stress distribution of 0.85 $f_c^{'}$ in the compression concrete slab. $f_c^{'}$ is minimum specified 28-day compressive strength of concrete. In positive flexure regions the contribution of reinforcement in the concrete slab is small and can be neglected. Table 6.4-3 summarizes calculations of M_p.

Table 6.4-3 Plastic Moment Calculation

Regions	Case and PNA	Condition and $\overline{Y}$	M_p
Positive Figure 6.4.2	I - In Web	$P_t + P_w \geq P_c + P_s + P_{rb} + P_{rt}$ $\overline{Y} = \left(\frac{D}{2}\right)\left[\frac{P_t - P_c - P_s - P_{rt} - P_{rb}}{P_w} + 1\right]$	$M_p = \frac{P_w}{2D}\left[\overline{Y}^2 + \left(D - \overline{Y}\right)^2\right] +$ $\left[P_s d_s + P_{rt} d_{rt} + P_{rb} d_{rb} + P_c d_c + P_t d_t\right]$
	II - In Top Flange	$P_t + P_w + P_c \geq P_s + P_{rb} + P_{rt}$ $\overline{Y} = \left(\frac{t_c}{2}\right)\left[\frac{P_w + P_t - P_s - P_{rt} - P_{rb}}{P_c} + 1\right]$	$M_p = \frac{P_c}{2t_c}\left[\overline{Y}^2 + \left(t_c - \overline{Y}\right)^2\right] +$ $\left[P_s d_s + P_{rt} d_{rt} + P_{rb} d_{rb} + P_w d_w + P_t d_t\right]$
	III- In Slab, Below P_{rb}	$P_t + P_w + P_c \geq \left(\frac{C_{rb}}{t_s}\right) P_s + P_{rb} + P_{rt}$ $\overline{Y} = \left(t_s\right)\left[\frac{P_w + P_c + P_t - P_{rt} - P_{rb}}{P_s}\right]$	$M_p = \left(\frac{\overline{Y}^2 P_s}{2t_s}\right) +$ $\left[P_{rt} d_{rt} + P_{rb} d_{rb} + P_c d_c + P_w d_w + P_t d_t\right]$
	IV - In Slab, Above P_{rb} Below P_{rt}	$P_t + P_w + P_c + P_{rb} \geq \left(\frac{C_{rt}}{t_s}\right) P_s + P_{rt}$ $\overline{Y} = \left(t_s\right)\left[\frac{P_{rb} + P_c + P_w + P_t - P_{rt}}{P_s}\right]$	$M_p = \left(\frac{\overline{Y}^2 P_s}{2t_s}\right) +$ $\left[P_{rt} d_{rt} + P_{rb} d_{rb} + P_c d_c + P_w d_w + P_t d_t\right]$
	V - In Slab, above P_{rt}	$P_t + P_w + P_c + P_{rb} + P_{rt} < \left(\frac{C_{rt}}{t_s}\right) P_s$ $\overline{Y} = \left(t_s\right)\left[\frac{P_{rb} + P_c + P_w + P_t + P_{rt}}{P_s}\right]$	$M_p = \left(\frac{\overline{Y}^2 P_s}{2t_s}\right) +$ $\left[P_{rt} d_{rt} + P_{rb} d_{rb} + P_c d_c + P_w d_w + P_t d_t\right]$
Negative Figure 6.4.3	I - In Web	$P_c + P_w \geq P_t + P_{rb} + P_{rt}$ $\overline{Y} = \left(\frac{D}{2}\right)\left[\frac{P_c - P_t - P_{rt} - P_{rb}}{P_w} + 1\right]$	$M_p = \frac{P_w}{2D}\left[\overline{Y}^2 + \left(D - \overline{Y}\right)^2\right] +$ $\left[P_{rt} d_{rt} + P_{rb} d_{rb} + P_t d_t + P_c d_c\right]$
	II - In Top Flange	$P_c + P_w + P_t \geq P_{rb} + P_{rt}$ $\overline{Y} = \left(\frac{t_t}{2}\right)\left[\frac{P_w + P_c - P_{rt} - P_{rb}}{P_t} + 1\right]$	$M_p = \frac{P_t}{2t_t}\left[\overline{Y}^2 + \left(t_t - \overline{Y}\right)^2\right] +$ $\left[P_{rt} d_{rt} + P_{rb} d_{rb} + P_w d_w + P_c d_c\right]$

$P_{rt} = F_{yrt} A_{rt}$; $P_s = 0.85 f'_c b_s t_s$; $P_{rb} = F_{yrb} A_{rb}$; $P_c = F_{yc} b_c t_t$; $P_w = F_{yw} D t_w$; $P_t = F_{yt} b_t t_t$;

f'_c = minimum specified 28-day compressive strength of concrete (ksi)
PNA = plastic neutral axis
A_{rb}, A_{rt} = reinforcement area of bottom and top layer in concrete deck slab (in.^{2})
F_{yrb}, F_{yrt} = yield strength of reinforcement of bottom and top layers (ksi)
b_c, b_t, b_s = width of compression, tension steel flange and concrete deck slab (in.)
t_c, t_t, t_w, t_s = thickness of compression, tension steel flange, web and concrete deck slab (in.)
F_{yt}, F_{yc}, F_{yw} = yield strength of tension flange, compression flange and web (ksi)

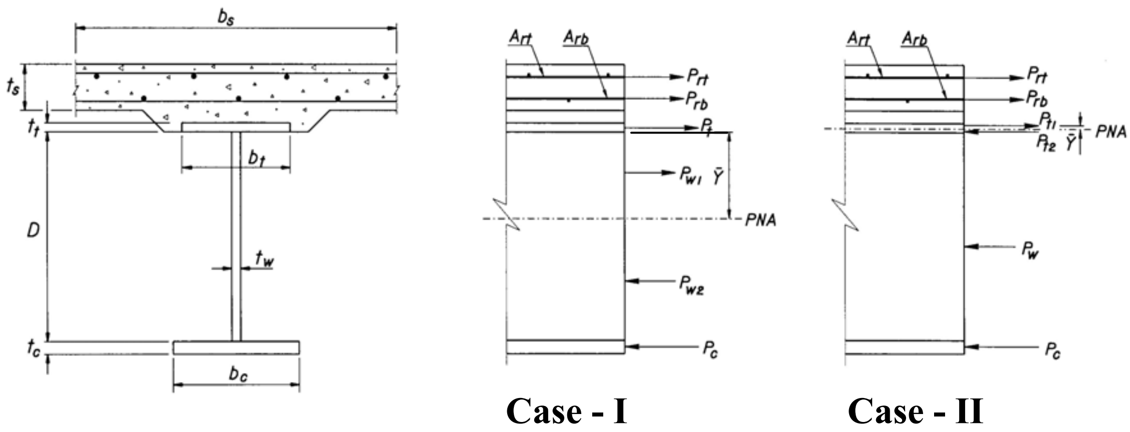


$$P_{c1} = \overline{Y} b_c F_{yc}; \qquad P_{c2} = \left(t_c - \overline{Y}\right) b_c F_{yc}$$

$$P_{w1} = \overline{Y} t_w F_{yw}; \qquad P_{w2} = \left(D - \overline{Y}\right) t_w F_{yw}$$

$$P_{t1} = \overline{Y} b_t F_{yt}; \qquad P_{t2} = \left(t_t - \overline{Y}\right) b_t F_{yt}$$

Figure 6.4-2 Plastic Moment Calculation Cases for Positive Flexure



$$P_{w1} = \overline{Y} t_w F_{yw}; \quad P_{w2} = \left(D - \overline{Y}\right) t_w F_{yw}$$

$$P_{t1} = \overline{Y} b_t F_{yt}; \quad P_{t2} = \left(t_t - \overline{Y}\right) t_t F_{yt}$$

Figure 6.4-3 Plastic Moment Calculation Cases for Negative Flexure

6.4.3 Steel Sections

The flexural resistance of a steel section (i.e., composite sections in negative flexure and noncomposite sections) is governed by three failure modes or limit states: yielding, flange local buckling and lateral-torsional buckling. The moment capacity depends on the yield strength of steel, the slenderness ratio of the compression flange, λ_f in terms of width-to-thickness ratio ($b_{fc}/2t_{fc}$) for local buckling and the unbraced length L_b for lateral-torsional buckling. Figure 6.4-4 shows dimensions of a typical I-girder. Figures 6.4-5 and 6.4-6 show graphically the compression flange local and lateral torsional buckling resistances, respectively. Calculations for nominal flexural resistances are illustrated in Table 6.4-4.

For sections in straight bridges satisfying the following requirements:

- $F_{yf} \leq 70$ ksi
- $\dfrac{2D_c}{t_w} \leq 5.7\sqrt{\dfrac{E}{F_{yc}}}$ (AASHTO 6.10.6.2.3-1)
- $\dfrac{I_{yc}}{I_{yt}} \geq 0.3$ (AASHTO 6.10.6.2.3-2)

The flexural resistance in term of moments may be determined by AASHTO Appendix A6, and may exceed the yield moment.

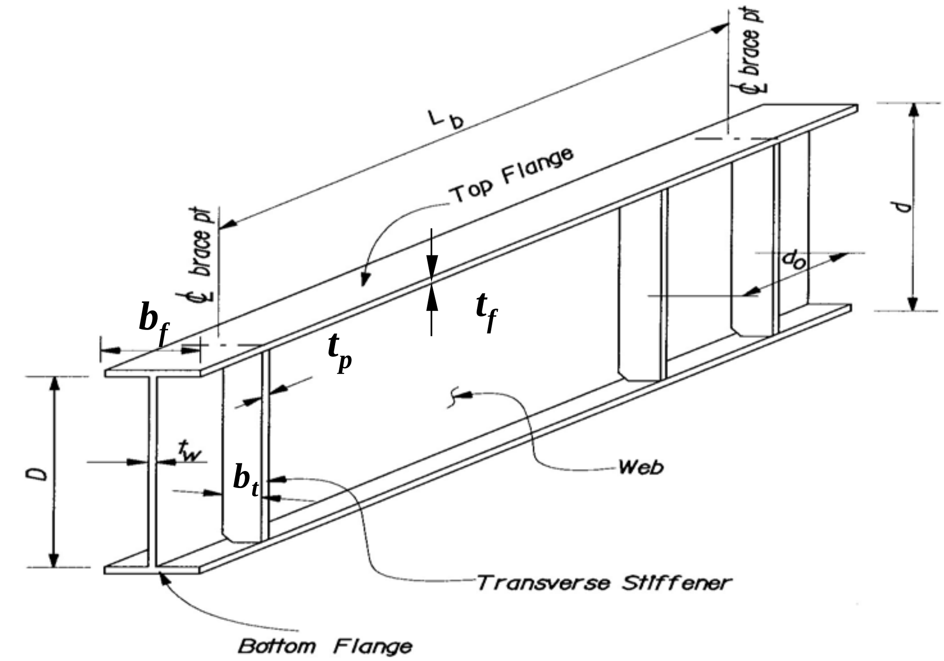


Figure 6.4-4 Dimensions of a Typical I-Girder

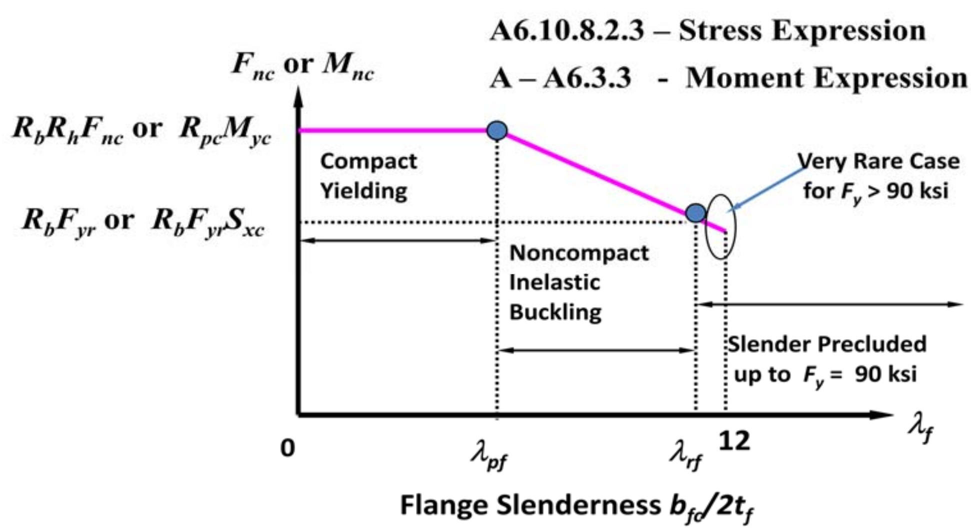


Figure 6.4-5 I-Section Compression-Flange Flexural Local-Buckling Resistance

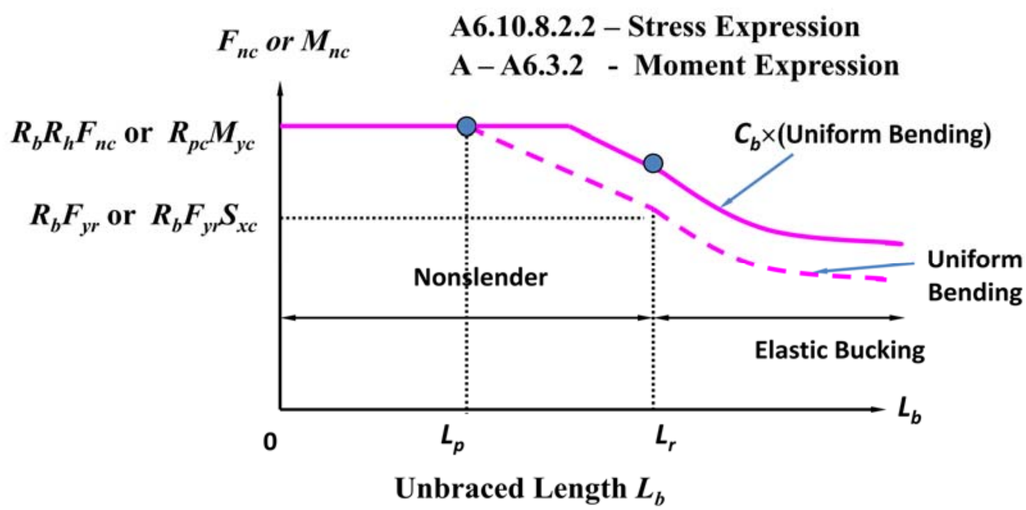


Figure 6.4-6 I-Section Compression-Flange Flexural Torsional Resistance

**Table 6.4-4 Nominal Flexural Resistance for Steel Sections
(Composite Sections in Negative Flexure and Noncomposite Section)
(Strength Limit State)**

Flange	Nominal Flexural Resistance
Compression	$F_{nc} = \text{smaller } [F_{nc(FLB)}, F_{nc(LTB)}]$ (AASHTO 6.10.8.2.1) $F_{nc(FLB)} = \begin{cases} R_b R_h F_{yc} & \text{for } \lambda_f \le \lambda_{pf} \\ \left[1-\left(1-\dfrac{F_{yr}}{R_h F_{yc}}\right)\left(\dfrac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] R_b R_h F_{yc} & \text{for } \lambda_f > \lambda_{pf} \end{cases}$ (AASHTO 6.10.8.2.2-1 &2) $F_{nc(LTB)} = \begin{cases} R_b R_h F_{yc} & \text{for } L_b \le L_p \\ C_b \left[1-\left(1-\dfrac{F_{yr}}{R_h F_{yc}}\right)\left(\dfrac{L_b - L_p}{L_r - L_p}\right)\right] R_b R_h F_{yc} \le R_b R_h F_{yc} & \text{for } L_b < L_p \le L_r \\ F_{cr} \le R_b R_h F_{yc} & \text{for } L_b > L_r \end{cases}$ (AASHTO 6.10.8.2.3-1, 2 &3)
Tension	$F_{nt} = R_h F_{yt}$ (AASHTO 6.10.8.3-1)

L_b = unbraced length of compression flange (in.)

L_p = limiting unbraced length to achieve $R_b R_h F_{yc} = 1.0 r_t \sqrt{E/F_{yc}}$ (AASHTO 6.10.8.2.3-4)

L_r = limiting unbraced length to achieve the onset of nominal yielding $= \pi r_t \sqrt{E/F_{yr}}$ (AASHTO 6.10.8.2.3-5)

λ_f = slenderess ratio for compression flange $= \dfrac{b_{fc}}{2t_{fc}}$ (AASHTO 6.10.8.2.2-3)

λ_{pf} = slenderess ratio for a compact compression flange $= 0.38\sqrt{\dfrac{E}{F_{yc}}}$ (AASHTO 6.10.8.2.2-4)

λ_{rf} = limiting slenderness ratio for a noncompact flange $= 0.56\sqrt{\dfrac{E}{F_{yc}}}$ (AASHTO 6.10.8.2.2-5)

F_{cr} = elastic lateral torsional buckling stress (ksi) $= \dfrac{C_b R_b \pi^2 E}{(L_b / r_t)^2}$ (AASHTO 6.10.8.2.3-8)

F_{yr} = smaller $\{0.7F_{yc},\ F_{yw}\} \ge 0.5F_{yc}$ (AASHTO 6.10.8.2.2)

C_b = moment gradient modifier

r_t = effective radius of gyration for lateral torsional buckling (in.)

6.5 SHEAR DESIGN

6.5.1 Design Requirements

For I-girder web panels, the following equation shall be satisfied.

$$V_u \leq \phi_c V_n \qquad \text{(AASHTO 6.10.9.1-1)}$$

where V_u is factored shear at the section under consideration (kip); V_n is nominal shear resistance (kip) and ϕ_c is resistance factor for shear = 1.0.

6.5.2 Nominal Shear Resistance

Similar to the flexural resistance, web shear resistance is also dependent on the slenderness ratio in terms of depth-to-thickness ratio (D/t_w).

For the web without transverse stiffeners, shear resistance is provided by the beam action of shearing yield or elastic shear buckling. For end panels of stiffened webs adjacent to simple support, shear resistance is limited to the beam action only.

$$V_n = V_{cr} = CV_p \qquad \text{(AASHTO 6.10.9.2-1; 9.3-1)}$$

$$V_p = 0.58 F_{yw} D t_w \qquad \text{(AASHTO 6.10.9.2-2)}$$

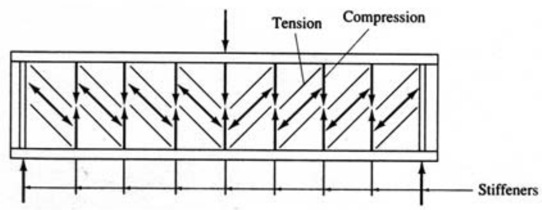
$$C = \begin{cases} 1.0 & \text{For } \dfrac{D}{t_w} \leq 1.12\sqrt{\dfrac{Ek}{F_{yw}}} \\ \dfrac{1.12}{(D/t_w)}\sqrt{\dfrac{Ek}{F_{yw}}} & \text{For } 1.12\sqrt{\dfrac{Ek}{F_{yw}}} < \dfrac{D}{t_w} \leq 1.40\sqrt{\dfrac{Ek}{F_{yw}}} \\ \dfrac{1.57}{(D/t_w)^2}\left(\dfrac{Ek}{F_{yw}}\right) & \text{For } \dfrac{D}{t_w} > 1.40\sqrt{\dfrac{Ek}{F_{yw}}} \end{cases}$$

(AASHTO 6.10.9.3.2-4,5,6)

$$k = 5 + \frac{5}{(d_o/D)^2} \qquad \text{(AASHTO 6.10.9.3.2-7)}$$

where d_o is transverse stiffener spacing (in.); C is ratio of the shear-buckling resistance to the shear yield strength; V_{cr} is shear-buckling resistance (kip) and V_p is plastic shear force (kip).

For interior web panels with transverse stiffeners, the shear resistance is provided by both the beam and the tension field actions as shown in Figure 6.5-1.



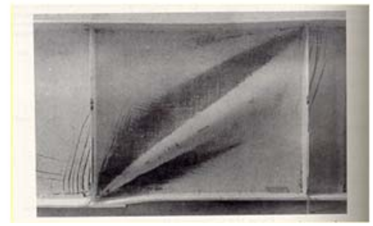


Figure 6.5-1 Tension Field Action

For $\frac{2Dt_w}{\left(b_{fc}t_{fc}+b_{ft}t_{ft}\right)} \leq 2.5$ (AASHTO 6.10.9.3.2-1)

$$V_n = V_p \left[C + \frac{0.87(1-C)}{\sqrt{1+\left(\frac{d_o}{D}\right)^2}} \right] \quad \text{(AASHTO 6.10.9.3.2-2)}$$

otherwise

$$V_n = V_p \left[C + \frac{0.87(1-C)}{\sqrt{1+\left(\frac{d_o}{D}\right)^2} + \frac{d_o}{D}} \right] \quad \text{(AASHTO 6.10.9.3.2-8)}$$

where b_{fc} and b_{ft} are full width of a compression and tension flange, respectively (in.); t_{fc} and t_{ft} are thickness of a compression and tension flange, respectively (in.); t_w is web thickness (in.) and d_o is transverse stiffener spacing (in.).

6.5.3 Transverse Stiffeners

Transverse intermediate stiffeners work as anchors for the tension field force so that post-buckling shear resistance can be developed. It should be noted that elastic web shear buckling cannot be prevented by transverse stiffeners. Transverse stiffeners are designed to (1) meet the slenderness requirement of projecting elements to prevent local buckling, (2) provide stiffness to allow the web to develop its post-buckling capacity, and (3) have strength to resist the vertical components of the diagonal stresses in the web.

6.5.4 Shear Connectors

To ensure a full composite action, shear connectors must be provided at the interface between the concrete slab and the steel to resist interface shear. Shear connectors are usually provided throughout the length of the bridge. If the longitudinal reinforcement in the deck slab is not considered in the composite section, shear connectors are not necessary in negative flexure regions. If the longitudinal reinforcement is included either additional connectors can be placed in the region of dead load contra-flexure points or they can be continued through the negative flexure region at maximum spacing. The fatigue and strength limit states must be considered in the shear connector design.

6.6 COMPRESSION DESIGN

6.6.1 Design Requirements

For axially loaded compression members, the following equation shall be satisfied:

$$P_u \le P_r = \phi_c P_n \tag{6.6-1}$$

where P_u is factored axial compression load (kip); P_r is factored axial compressive resistance (kip); P_n is nominal compressive resistance (kip) and ϕ_c is resistance factor for compression = 0.9.

For members subjected to combined axial compression and flexure, the following interaction equation shall be satisfied:

For $\dfrac{P_u}{P_r} < 0.2$

$$\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0 \tag{AASHTO 6.9.2.2-1}$$

For $\dfrac{P_u}{P_r} \ge 0.2$

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0 \tag{AASHTO 6.9.2.2-2}$$

where M_{ux} and M_{uy} are factored flexural moments (second-order moments) about the x-axis and y-axis, respectively (kip-in.); M_{rx} and M_{ry} are factored flexural resistance about the x-axis and y-axis, respectively (kip-in.).

Compression members shall also meet the slenderness ratio requirements, $Kl/r \le 120$ for primary members, and $Kl/r \le 140$ for secondary members. K is effective length factor; l is unbraced length (in.) and r is minimum radius of gyration (in.).

6.6.2 Axial Compressive Resistance

For steel compression members with non-slender elements, axial compressive resistance equations specified in the AASHTO (2012) are identical to the column design equations in AISC (2010).

For $\dfrac{P_e}{P_o} \geq 0.44$

$$P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] P_o \qquad \text{(AASHTO 6.9.4.1.1-1)}$$

For $\dfrac{P_e}{P_o} < 0.44$

$$P_n = 0.877 P_e \qquad \text{(AASHTO 6.9.4.1.1-2)}$$

$$P_e = \frac{\pi^2 E}{\left(\dfrac{Kl}{r_s}\right)^2_{eff}} \qquad \text{(AASHTO 6.9.4.1.2-1)}$$

in which

$$P_o = QF_y A_g$$

$$P_e = \frac{\pi^2 E}{\left(\dfrac{Kl}{r_s}\right)^2_{eff}} A_g \qquad \text{(AASHTO 6.9.4.1.2-1)}$$

where A_g is gross cross section area (in.^{2}); K is effective length factor in the plane of buckling; l is unbraced length in the plan of buckling (in.); r_s is radius of gyration about the axis normal to the plane of buckling (in.); Q is slender element reduction factor determined as specified in AASHTO Article 6.9.4.2.

6.7 TENSION DESIGN

6.7.1 Design Requirements

For axially loaded tension members, the following equation shall be satisfied:

$$P_u \leq P_r \qquad (6.7\text{-}1)$$

where P_u is factored axial tension load (kip) and P_r is factored axial tensile resistance (kip).

For members subjected to combined axial tension and flexure, the following interaction equation shall be satisfied:

For $\dfrac{P_u}{P_r} < 0.2$

$$\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0 \qquad \text{(AASHTO 6.8.2.3-1)}$$

For $\dfrac{P_u}{P_r} \geq 0.2$

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0 \qquad \text{(AASHTO 6.8.2.3-2)}$$

where M_{ux} and M_{uy} are factored flexural moments about the *x*-axis and *y*-axis, respectively (kip-in.); M_{rx} and M_{ry} are factored flexural resistance about the *x*-axis and *y*-axis, respectively (kip-in.).

Tension members shall also meet the slenderness ratio requirements, $l/r \leq 140$ for primary members subjected to stress reversal, $l/r \leq 200$ for primary members not subjected to stress reversal, and $l/r \leq 240$ for secondary members.

6.7.2 Axial Tensile Resistance

For steel tension members, axial tensile resistance equations are smaller of yielding on the gross section and fracture on the net section as follows:

Yielding in gross section:

$$P_r = \phi_y P_{ny} = \phi_y F_y A_g \qquad \text{(AASHTO 6.8.2.1-1)}$$

Fracture in net section:

$$P_r = \phi_u P_{nu} = \phi_u F_u A_n U \qquad \text{(AASHTO 6.8.2.1-2)}$$

where P_{ny} is nominal tensile for yielding in gross section (kip); P_{nu} is nominal tensile for fracture in net section (kip); A_n is net cross section area (in.2); F_u is specified minimum tensile strength (ksi); U is reduction factor to account for shear leg; ϕ_y is resistance factor for yielding of tension member = 0.95; ϕ_u is resistance factor for fracture of tension member = 0.8.

6.8 FATIGUE DESIGN

There are two types of fatigue: load and distortion induced fatigue. The basic fatigue design requirement for load-induced fatigue is limiting live load stress range to fatigue resistance for each component and connection detail. Distortion-induced fatigue usually occurs at the web near a flange due to improper detailing. The design requirement for distortion-induced fatigue is to follow proper detailing practice to provide sufficient load paths. For load-induced fatigue consideration, the most common types of components and details in a typical I- girder are (AASHTO Table 6.6.1.2.3-1) listed in Table 6.8-1.

Table 6.8-1 I-Section Flexural Design Equations (Strength Limit State)

Type of Details		Category (AASHTO Table 6.6.1.2.3-1)
1	Base metal and weld metal at full-penetration groove-welded splices	B
2	Base metal at gross section of high-strength bolted slip-critical connections (bolt gusset to flange)	B
3	Base metal at fillet-welded stud- type shear connectors	C
4	Base metal at toe of transverse stiffener-to-flange and transverse stiffener-to-web welds	C′

Nominal fatigue resistance as shown in Figure 6.8-1 (AASHTO, 2012) is calculated as follows:

For infinite fatigue life ($N > N_{TH}$)

$$\left(\Delta F_n\right) = \left(\Delta F\right)_{TH} \qquad \text{(AASHTO 6.1.2.5-1)}$$

For finite fatigue life ($N \leq N_{TH}$)

$$\left(\Delta F_n\right) = \left(\frac{A}{N}\right)^{\frac{1}{3}} \qquad \text{(AASHTO 6.6.1.2.5-2)}$$

in which:

$$N = (365)(75)n(ADTT)_{SL} \qquad \text{(AASHTO 6.6.1.2.5-3)}$$

$$N_{TH} = \frac{A}{\left[\left(\Delta F\right)_{TH}\right]^3} \qquad \text{(CA 6.6.1.2.3-2)}$$

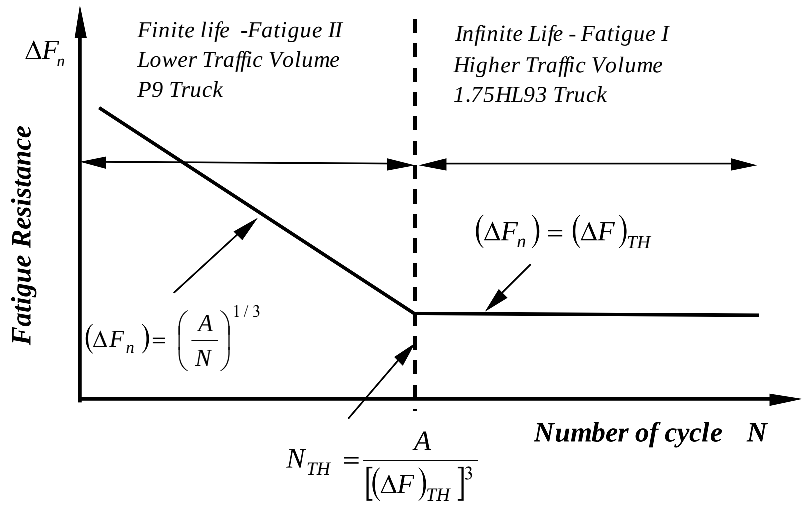


Figure 6.8-1 Fatigue Resistance

where A is a constant depending on detail category as specified in AASHTO Table 6.6.1.2.5-1, and $(\Delta F)_{TH}$ is the constant-amplitude fatigue threshold taken from AASHTO Table 6.6.1.2.5-3. N_{TH} is minimum number of stress cycles corresponding to constant-amplitude fatigue threshold, $(\Delta F)_{TH}$, as listed in CA Table 6.6.1.2.3-2.

$$ADTT_{SL} = p(ADTT) \qquad \text{(AASHTO 3.6.1.4.2-1)}$$

where p is fraction of truck traffic in a single lane (AASHTO Table 3.6.1.4.2-1) = 0.8 for three or more lanes traffic, N is the number of stress-range cycles per truck passage = 1.0 for the positive flexure region for span > 40 ft. (CA Table 6.6.1.2.5-2). *ADTT* is the number of trucks per day in one direction averaged over the design life and is specified in CA 3.6.1.4.2.

Fatigue I: $ADTT = 2500, \quad N = (365)(75)(1.0)(0.8)(2500) = 0.5475(10)^8 > N_{TH}$

Fatigue II: $ADTT = 20, \quad N = (365)(75)(1.0)(0.8)(20) = 438{,}000 < N_{TH}$

The nominal fatigue resistances for typical Detail Categories in an I-girder are summarized in Table 6.8-2.

Table 6.8-2 Nominal Fatigue Resistance

Detail Category	Constant –A ($\times\ 10^8$) (ksi^3)	Fatigue II $\left(\Delta F_n\right) = \left(\dfrac{A}{N}\right)^{\frac{1}{3}}$ (ksi)	Fatigue I $\left(\Delta F_n\right) = \left(\Delta F\right)_{TH}$ (ksi)
B	120.0	30.15	16.0
C	44.0	21.58	10.0
C′	44.0	21.58	12.0
E	11.0	13.59	4.5

6.9 SERVICEABILITY STATES

The service limit state design is intended to control the elastic and permanent deformations, which would affect riding ability. For steel girder, vehicular live load deflection may be limited to $L/800$ by AASHTO 2.5.2.6.

Based on past successful practice of the overload check in the AASHTO *Standard Specifications* (AASHTO, 2002) to prevent the permanent deformation due to expected severe traffic loadings, AASHTO 6.10.4 requires that for SERVICE II load combination, flange stresses in positive and negative bending without considering flange lateral bending, f_f shall meet the following requirements:

For the top steel flange of composite sections

$$f_f \leq 0.95 R_h F_{yf} \qquad \text{(AASHTO 6.10.4.2.2-1)}$$

For the bottom steel flange of composite sections

$$f_f + \frac{f_l}{2} \leq 0.95 R_h F_{yf} \qquad \text{(AASHTO 6.10.4.2.2-2)}$$

For both steel flanges of noncomposite sections

$$f_f + \frac{f_l}{2} \leq 0.8\, R_h F_{yf} \qquad \text{(AASHTO 6.10.4.2.2-3)}$$

For compact composite sections in positive flexure in shored construction, longitudinal compressive stress in concrete deck without considering flange lateral bending, f_c, shall not exceed $0.6 f_c'$ where f_c' is minimum specified 28-day compressive strength of concrete (ksi).

Except for composite sections in positive flexure satisfying $D/t_w \leq 150$ without longitudinal stiffeners, all sections shall satisfy

$$f_c \leq F_{crw} \qquad \text{(AASHTO 6.10.4.2.2-4)}$$

6.10 CONSTRUCTIBILITY

An I-girder bridge constructed in unshored conditions shall be investigated for strength and stability for all critical construction stages, using the appropriate strength load combination discussed in Chapter 3. All calculations shall be based on the non-composite steel section only.

AASHTO Article 6.10.3 requires checking the following requirements:

- *Compression Flange*

  For discretely braced flange (AASHTO 6.10.3.2.1)

$$f_{bu} + f_l \le \phi_f R_h F_{yc} \qquad \text{(AASHTO 6.10.3.2.1-1)}$$

$$f_{bu} + \frac{1}{3} f_l \le \phi_f F_{nc} \qquad \text{(AASHTO 6.10.3.2.1-2)}$$

$$f_{bu} \le \phi_f F_{crw} \qquad \text{(AASHTO 6.10.3.2.1-3)}$$

where f_{bu} is flange stress calculated without consideration of the flange lateral bending (ksi); F_{crw} is nominal bending stress determined by AASHTO 6.10.1.9.1-1 (ksi).

For sections with compact and noncompact webs, AASHTO Equation 6.10.3.2.1-3 shall not be checked. For sections with slender webs, AASHTO Equation 6.10.3.2.1-1 shall not be checked when f_l is equal to zero.

For continuously braced flanges

$$f_{bu} \le \phi_f R_h F_{yc} \qquad \text{(AASHTO 6.10.3.2.3-1)}$$

- *Tension Flange*

  For discretely braced flange

$$f_{bu} + f_l \le \phi_f R_h F_{yc} \qquad \text{(AASHTO 6.10.3.2.1-1)}$$

For continuously braced flanges

$$f_{bu} \le \phi_f R_h F_{yt} \qquad \text{(AASHTO 6.10.3.2.3-1)}$$

- *Web*

$$V_u \le \phi_v V_{cr} \qquad \text{(AASHTO 6.10.3.3-1)}$$

where V_u is the sum of factored dead loads and factored construction load applied to the non-composite section (AASHTO 6.10.3.3) and V_{cr} is shear buckling resistance (AASHTO 6.10.9.3.3-1).

NOTATION

A	=	fatigue detail category constant
$ADTT$	=	average daily truck traffic in one direction over the design life
$ADTT_{SL}$	=	single lane *ADTT* life
A_g	=	gross cross section area (in.^{2})
A_n	=	net cross section area (in.^{2})
A_{rb}	=	reinforcement area of bottom layer in concrete deck slab (in.^{2})
A_{rt}	=	reinforcement area of top layer in concrete deck slab (in.^{2})
b_c	=	width of compression steel flange (in.)
b_f	=	full width of the flange (in.)
b_{fc}	=	full width of a compression flange (in.)
b_{ft}	=	full width of a tension flange (in.)
b_s	=	width of concrete deck slab (in.)
b_t	=	width of tension steel flange (in.)
C	=	ratio of the shear-buckling resistance to the shear yield strength
C_b	=	moment gradient modifier
D	=	web depth (in.)
D_{cp}	=	web depth in compression at the plastic moment (in.)
D_p	=	distance from the top of the concrete deck to the neutral axis of the composite sections at the plastic moment (in.)
D_t	=	total depth of the composite section (in.)
d	=	total depth of the steel section (in.)
d_o	=	transverse stiffener spacing (in.)
E	=	modulus of elasticity of steel (ksi)
F_{cr}	=	elastic lateral torisonal buckling stress (ksi)
F_{crw}	=	nominal bend-buckling resistance of webs (ksi)
F_{exx}	=	classification strength specified of the weld metal
F_{nc}	=	nominal flexural resistance of the compression flange (ksi)
F_{nt}	=	nominal flexural resistance of the tension flange (ksi)
F_{yc}	=	specified minimum yield strength of a compression flange (ksi)
F_{yf}	=	specified minimum yield strength of a flange (ksi)

F_{yr} = compression-flange stress at the onset of nominal yielding including residual stress effects, taken as the smaller of $0.7F_{yc}$ and F_{yw} but not less than $0.5F_{yc}$ (ksi)

F_{yrb} = specified minimum yield strength of reinforcement of bottom layers (ksi)

F_{yrt} = specified minimum yield strength of reinforcement of top layers (ksi)

F_{ys} = specified minimum yield strength of a stiffener (ksi)

F_{yt} = specified minimum yield strength of a tension flange (ksi)

F_{yw} = specified minimum yield strength of a web (ksi)

F_{yu} = specified minimum tensile strength of steel (ksi)

f_{bu} = flange stress calculated without consideration of the flange lateral bending (ksi)

f_c = longitudinal compressive stress in concrete deck without considering flange lateral bending (ksi)

f_f = flange stresses without considering flange lateral bending (ksi)

f_s = maximum flexural stress due to Service II at the extreme fiber of the flange (ksi)

f_{sr} = fatigue stress range (ksi)

f'_c = minimum specified 28-day compressive strength of concrete (ksi)

I = moment of inertia of a cross section (in.^{4})

I_{yc} = moment of inertia of the compression flange about the vertical axis in the plane of web (in.^{4})

I_{yt} = moment of inertia of the tension flange about the vertical axis in the plane of web (in.^{4})

K = effective length factor of a compression member

L = span length (ft)

L_b = unbraced length of compression flange (in.)

L_p = limiting unbraced length to achieve $R_bR_hF_{yc}$ (in.)

L_r = limiting unbraced length to onset of nominal yielding (in.)

l = unbraced length of member (in.)

M_{AD} = additional live load moment to cause yielding in either steel flange applied to the short-term composite section and can be obtained from the following equation (kip- in.)

M_{D1} = moment due to factored permanent loads applied to the steel section alone (kip-in.)

M_{D2}	=	moment due to factored permanent loads such as wearing surface and barriers applied to the long-term composite section (kip-in.)
M_p	=	plastic moment (kip-in.)
M_n	=	nominal flexural resistance of the section (kip-in.)
M_{rx}, M_{ry}	=	factored flexural resistance about the x-axis and y-axis, respectively (kip-in.)
M_u	=	bending moment about the major axis of the cross section (kip-in.)
M_{ux}, M_{uy}	=	factored flexural moments about the x-axis and y-axis, respectively (kip-in.)
M_y	=	yield moment (kip-in.)
N	=	number of cycles of stress ranges
N_{TH}	=	minimum number of stress cycles corresponding to constant-amplitude fatigue threshold, $(\Delta F)_{TH}$
n	=	number of stress-range cycles per truck passage
P_u	=	factored axial load (kip)
P_r	=	factored axial resistance (kip)
p	=	fraction of truck traffic in a single lane
Q	=	slender element reduction factor
R_h	=	hybrid factor
R_b	=	web load-shedding factor
R	=	radius of gyration
r_t	=	effective radius of gyration for lateral torsional buckling (in.)
S_{LT}	=	elastic section modulus for long-term composite sections, respectively (in.^{3})
S_{NC}	=	elastic section modulus for steel section alone (in.^{3})
S_{ST}	=	elastic section modulus for short-term composite section (in.^{3})
S_{xt}	=	elastic section modulus about the major axis of the section to the tension flange taken as M_{yt}/F_{yt} (in.^{3})
t_c	=	thickness of compression steel flange (in.)
t_f	=	thickness of the flange (in.)
t_{fc}	=	thickness of a compression flange (in.)
t_{ft}	=	thickness of a tension flange (in.)
t_t	=	thickness of tension steel flange (in.)
t_w	=	thickness of web (in.)
t_s	=	thickness of concrete deck slab (in.)
V_{cr}	=	shear-buckling resistance (kip)

V_n	=	nominal shear resistance (kip)
V_p	=	plastic shear force (kip)
V_u	=	factored shear (kip)
λ_f	=	slenderness ratio for compression flange = $b_{fc}/2t_{fc}$
λ_{pf}	=	limiting slenderness ratio for a compact flange
λ_{rf}	=	limiting slenderness ratio for a noncompact flange
$(\Delta F)_{TH}$	=	constant-amplitude fatigue threshold (ksi)
$(\Delta F)_n$	=	fatigue resistance (ksi)
ϕ_f	=	resistance factor for flexure = 1.0
ϕ_v	=	resistance factor for shear = 1.0
ϕ_c	=	resistance factor for axial compression = 0.9
ϕ_u	=	resistance factor for tension, fracture in net section = 0.8
ϕ_y	=	resistance factor for tension, yielding in gross section = 0.95

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